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A resilient shallow foundation system in coastal zone with liquefaction and lateral spreading potential

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ABSTRACT: Piled foundations have historically been employed to support buildings in coastal environments to mitigate against the risk of flooding and sea erosion. In earthquake prone areas however and based on experience around Christchurch, piled foundations are usually difficult to repair following a damaging earthquake and well-designed resilient shallow foundation solutions are preferred where practicable.

The paper describes a composite shallow foundation system comprising a reinforced concrete slab over a reinforced gravel mattress used to support an architecturally designed surf life saving club building complex in a coastal environment subject to liquefaction, lateral spreading, flooding and sea erosion. It also discusses geotextile sand containers used as 'sleeping defences' for coastal erosion protection and their benefits compared to other traditional sea defence structures where there is liquefaction potential hazard. The ground conditions and site hazards are discussed for contextual purposes together with feasible options considered. The paper highlights, for this particular site and ground conditions, that the composite shallow foundation system is more resilient and cost effective when compared to an equivalent piled foundation system.

1 INTRODUCTION

Piled foundations have historically been employed to support buildings in coastal environments to mitigate against the risk of flooding and sea erosion. This is mainly because piles can easily be used to provide required elevated floor levels to avoid flooding, be designed for loss of soil support due to erosion and/scour, and provide free passage of floodwaters/waves beneath the raised floor which minimises damage to structural elements below flood levels. The piles can also be cost effectively driven in favourable ground conditions potentially where embedment into rock and/or dense gravel is not required.

In earthquake prone areas and particularly where there is a risk of liquefaction, lateral spreading and associated cyclic ground displacements, there is added kinematic soil loading of the piles usually resulting in more costly foundation systems from both a design and construction perspective. In addition to the capital cost, and based on experience from Christchurch, repairing piled foundations following a damaging earthquake is difficult and usually results in costly foundation rebuilds even when superstructures have sustained minor damage. As a consequence, well designed resilient shallow foundation systems are preferred where practicable as they are easily re-levellable following a major earthquake event.

This paper describes attributes of a resilience shallow foundation system for a replacement Sumner Surf Life Saving Club (SLSC) building in Christchurch to mitigate against liquefaction and lateral spreading hazard, coastal erosion, and flooding. The shallow foundation system was adopted in lieu of deep piles that were considered difficult to repair in a damaging event and also costly for the ground conditions. The new building complex was a replacement of the original 1950 Sumner SLSC building that was damaged beyond economic repair during the Canterbury Earthquake Sequence between September 2010 and December 2011.

2 THE SITE AND GROUND CONDITIONS

The site is located at 301 Main Road, Sumner Christchurch and slopes from about 12.7mRL at the road level to 11.2mRL to the landward side of the dune sands (see Figure 1 below). An approximate 0.6m high dune separates the site from Sumner beach, and there is a near vertical 15m to 20m high rock cliff to the south of the site across the road.

Figure 1 – Aerial Photographs showing site location (on the left) and site detail (on the right) including location of CPT investigations. Image background sourced from LINZ Crown Copyright Reserved.

A review of the regional geology (Brown and Weeber, 1992) and ground investigations including the two Cone Penetration Tests CPT1 and CPT2 in Figure 1 around the site indicate the following:

- The site is underlain by *'Sand of active dunes and present day beaches, ps'* overlying *'Dominantly sand of fixed and semi-fixed dunes and beaches, ch'*.
- The sand overlies volcanic rock of the *Mt Pleasant Formation (Ip)* and/or *Lyttleton Volcanic Group (I)* shown to outcrop within 20m of the southern site boundary.
- The onsite investigation logs indicate the sand as loose to medium dense in the upper 1m profile, and medium dense to very dense between 1m and the 12m depth investigated.
- The logs of CPT1 and CPT2, and accompanying Dynamic Penetrometer Super Heavy adjacent to the CPT positions indicate that the underlying rock slopes from about 5.5m depth at CPT1 location to 12.5m depth at CPT2 location over a distance of less than 50m.
- Groundwater table from 0.5m to below 1.6m depth depending on elevation around the site and seasonal and tidal variations. Groundwater level at approximate ground level (11.30mRL) towards the northern end was assumed for design purposes based on Christchurch City Council recommended flood level. For construction purposes, a groundwater table at 0.8m depth at the northern end (approximately 10.4mRL) was considered reasonable.

A coastal hazard assessment undertaken by Single (2012) indicates that there have been three periods of erosion and three periods of accretion between 1849 and 1980 with the high tide shoreline close to the cliffs between 1849 and 1907. Also of particular note is a major erosion event in the late 1970s to early 1980s which lowered the beach and undermined the seaward garage of the Sumner SLSC building at the time, and reported cases of wave-run up reaching the SCSC building since dune stabilisation measures were undertaken.

3 SEISMICITY

Christchurch was affected by four major earthquake events between September 2010 and December 2011 with Moment Magnitude between M_w5.9 and M_w7.1. The site is 6.6km from the epicentre of the M_w6.2 Christchurch Earthquake of 22 February 2011 and 1.5km from the epicentre of the M_w6.0 major aftershock of 13 June 2011 (GNS, 2015). The estimated peak ground acceleration at the site during the 22 February 2011 and 13 June 2011 based on median values from Bradley and Hughes (2012a, b) was 0.53g and 0.43g respectively.

Using the Idriss and Boulanger (2008) magnitude scaling factor, the site is considered to have undergone through four SLS level earthquake events (M_w7.5, 0.13g PGA) during the 4 September 2010, 22 February 2011, 13 June 2011, and 23 December 2011. The 22 February 2011 was close to a ULS level earthquake event (M_w7.5, 0.4g PGA).

A review of the Canterbury Geotechnical Database (CGD, 2015) and discussions with Sumner SLSC members indicate that there was significant ground and building damage including cracking in Main Road across the site and sand boils within the grassed areas.

4 THE PROPOSED BUILDING COMPLEX

The new building complex comprises an architecturally designed single storey (except two storey life guard observation tower) building across an approximate 550m^2 footprint and additional 200m^2 timber deck with associated canopy structures (see Figure 2 below). The structural form comprises timber and steel frame with precast concrete panels in places.

Figure 2 – Architectural Visualisation of proposed building complex from the beach end

5 LIQUEFACTION HAZARD ASSESSMENT

A liquefaction hazard assessment was undertaken based on Idriss and Boulanger (2008) to better understand the liquefaction and lateral spreading potential at the site. The assessment was based on the CPT1 and CPT2 on Figure 1, and showed the following:

- The site is prone to liquefaction in both an SLS and ULS design earthquake event due to the presence of potentially liquefiable sand layers at the site. The liquefiable layers are generally below 9m depth under SLS level event but are spread throughout the depth profile under a ULS event (see Figure 3). Minor liquefiable layers were assessed in the top 1m depth under SLS.
- The calculated liquefaction induced settlements could be up to 100mm and 200mm in an SLS and ULS earthquake event respectively.
- The differential free field liquefaction induced settlement could be up to 100mm in either SLS or ULS event because of the variable depth to rock across the site (i.e. 5m inferred from CPT1 and 12m inferred from CPT2).
- The assessed liquefaction induced damage potential to shallow founded structures is considered low under SLS level earthquake due to the thick non-liquefiable crust provided that the near surface loose sand is densified.
- Major surface expression of liquefaction and damage to structures is likely in a ULS level

earthquake event.

- Because of the gentle slope towards the sea, there is potential for both global lateral spreading and lateral stretch across proposed building in a ULS earthquake event due to presence of a near surface liquefiable layer across the site. A maximum lateral stretch across the building footprint of 100mm was considered reasonable due to absence of major cracks across the site following the 22 February 2011 earthquake which was close to a ULS design earthquake.
- Due to presence of intermediate non-liquefiable layers under a ULS design event, design of any deep piles at the site should allow for kinematic loading of the piles from the soils during shaking (cyclic displacements) and/or lateral spreading.

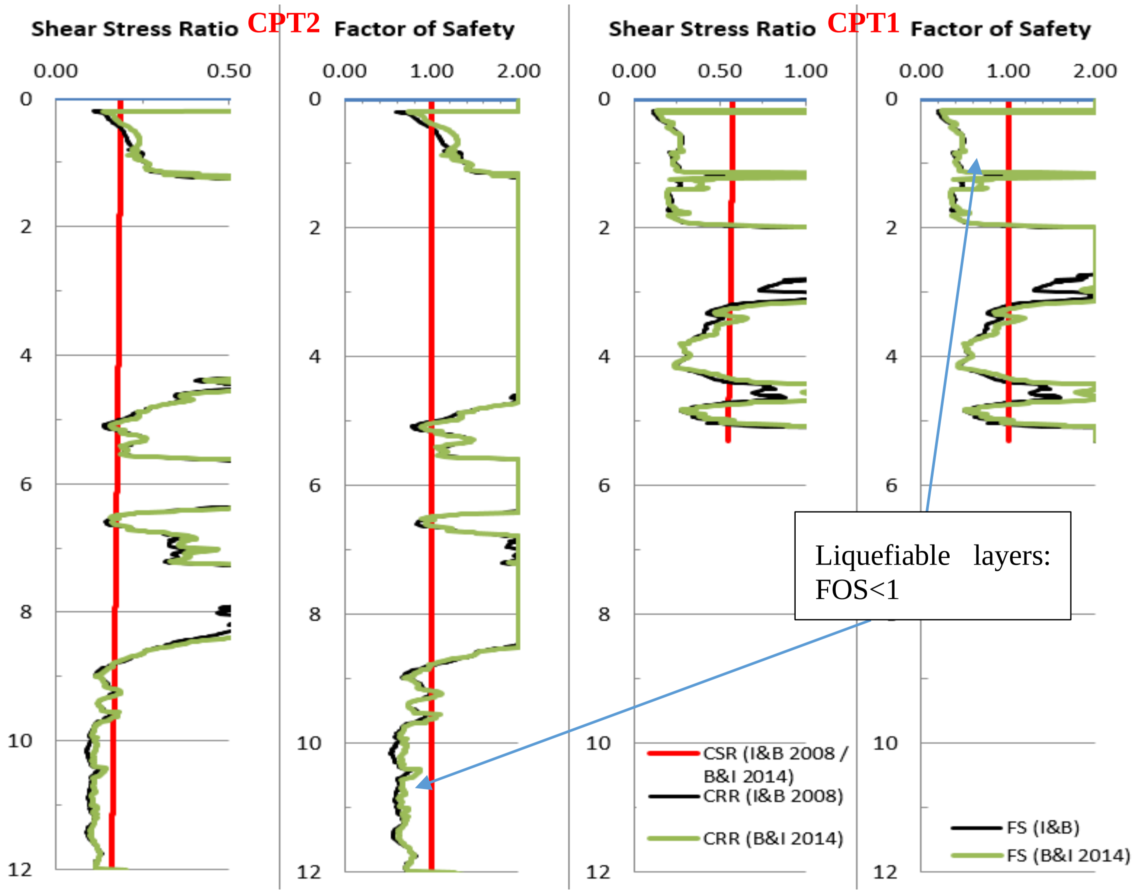

Figure 3: Results of Liquefaction Analysis under ILS earthquake event (Mw7.5, PGA 0.35g) for CPT 1 and CPT2

A review of the analysis has been completed for this paper based on Boulanger and Idriss (2014) and similar conclusions were reached.

6 DEVELOPMENT OF AN APPROPRIATE FOUNDATION SOLUTION

Based on discussions above, the main geotechnical hazards to the proposed development were considered to be coastal flooding, erosion and/or scour following a major storm event, damage to foundation and/or structural elements, liquefaction induced differential settlement, and lateral spreading of the site and/or lateral stretch across the building footprint.

Following a detailed options assessment exercise, the foundation options in Table 1 below were considered feasible for the site. Based on the comparisons of the three feasible options at the site as presented in Table 1 and in consultation with the Client, Option 3 was considered to be more appropriate in terms of resilience following a major earthquake event. The option was also more cost effective compared to the other two options. However, the option was still vulnerable to increased damage in a major erosion/scour event due to the potential to undermine the foundations.

Option 3 was therefore modified to produce a composite shallow foundation system described in Section 7 that was resilient to both coastal and liquefaction hazards.

Table 1 – Summary of Feasible Options Considered

Foundation Option	Technical Risks/Disadvantages	Benefits/Opportunities
Option 1 – Deep piles	• Larger diameter piles (than normal for a single storey building) to resist additional lateral loads on piles from kinematic loading. • Piles have to be embedded into underlying rock beyond the liquefiable layers to provide lateral fixity. • Range of pile diameters required with shorter slender piles at the road end where depth to rock is small and larger diameter piles at the seaward end where depth to rock is greater to give a balanced load deflection response. • Rock depth likely to be variable requiring more extensive investigations to confirm depth and strength parameters. • Piled buildings generally difficult to repair following a damaging earthquake compared with shallow foundations. • Susceptible to corrosion if steel or concrete.	• Proven foundation system in a range of coastal hazard zones. • Piles adaptable to varying degree of scour levels. • Large diameter piles required due to scour, and liquefaction related hazards likely to be resistant to forces from storm water and wave/flood borne debris. • Open space between the piles and raised floor allows relatively free passage of water thus minimising the impact to structure from waves and waterborne debris. • No imported fill required to achieve elevated floor levels above flood levels.
Option 2 – Concrete raft slab supporting short raised piles/columns similar to MBIE Option 2 Foundation System	• Susceptible to being undermined due to erosion/scour unless founded below scour line. • Short piles likely to require significant bracing to resist impact loads. • Bracing prevents free flow of water and waterborne debris and is thus susceptible to damage. • Sloping ground could require some form of retaining walls towards the raised road level. • Susceptible to corrosion if steel or concrete.	• Easily re-levellable following an earthquake event. • Likely to be more economic when compared to deep piles. • Earthquake resilience easily improved by incorporation of reinforced geogrid raft. • Limited imported fill required to achieve elevated floor levels above flood levels.
Option 3 – Slab on grade foundation on geogrid reinforced gravel raft	• Susceptible to being undermined due to erosion/scour unless founded below scour line. • Obstructs free flow of storm surges/flood water with potential increase in impact loading to foundation system. • Increased volume of fill to achieve elevated floor levels above flood levels.	• Reduced corrosion risk if structural members are above flood levels. • Relatively ‘inert’ geotextiles and geogrids can be adopted for gravel raft. • Easily re-levellable following damaging earthquake. • Reinforced gravel raft mutes effects of liquefaction induced differential settlement to structure.

7 THE COMPOSITE RESILIENT SHALLOW FOUNDATION SYSTEM

The key attributes of the composite foundation system adopted are shown on Figure 4 with benefits of each element presented in the sections below. It is noted that the sketch is based on the design drawings and minor changes were undertaken during construction to limit the loads on the Elcorock® Containers.

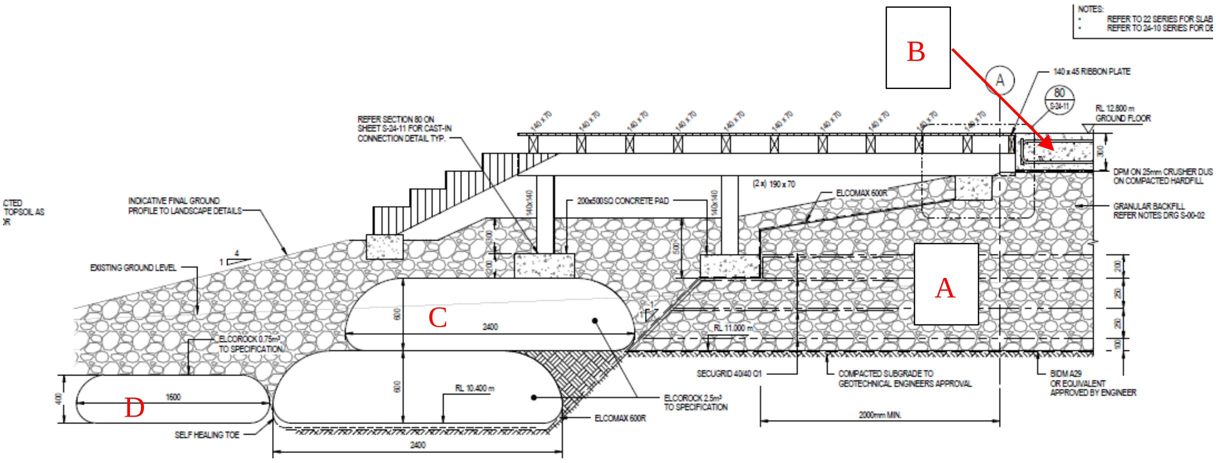

Figure 4 – Typical Seaward Section through Composite Shallow Foundation System at Design Stage

7.1 Element A – Reinforced Geogrid Gravel Raft

The reinforced geogrid gravel raft comprised Bidim A29 basal geotextile, three layers of Secugrid 40/40 Q1, and compacted crushed well graded CAP40 aggregate meeting particular grading requirements. The reinforced geogrid gravel raft together with an overlying reinforced concrete slab (Element B in Figure 4) provided the main composite foundation system for the building to mitigate against liquefaction induced damage.

The minimum thickness of the geogrid raft was chosen to provide optimal balance between amount of natural soil to be removed from site, working mainly above the water table, providing sufficient thickness of geogrid reinforced mattress to minimise the effect of differential settlement, and the need to raise site levels to meet the recommended flood levels.

The gravel raft was extended at least 2m beyond the foundation slab on the seaward side for added protection against wave action/erosion while 1m beyond the foundation slab was considered appropriate on all the other sides.

The added benefit of the reinforced geogrid gravel raft was the ability to densify near surface loose sand soils during compaction.

7.2 Element B – Reinforced Concrete Slab

The structural foundation slab transfers the loads from the superstructure to the underlying reinforced gravel raft and was designed to cantilever 2m at the edges in the event that the gravel raft is undermined during a major storm event and/or damaging earthquake. Designing for a 2m cantilever also meant that the extent of the gravel raft beyond the structural foundation slab could be reduced allowing more flexibility in landscaping beyond the gravel raft.

7.3 Element C – 2.5m^3 Elcorock® Containers

The 2.5m^3 Elcorock® Containers were used as the primary line of defence against a significant erosion event undermining the reinforced gravel raft mattress and were founded below known historical erosion levels. The containers were preferred to other traditional seawall defences because they were cost effective, could easily be adaptable in the future if length or height needed to be increased, locally sourced sand could be re-used, and they provide more flexibility to landscape designers. At Sumner, they have been designed to be covered with either natural dune sand and/or compacted gravel depending on landscape requirements.

7.4 **Element D – 0.75m³ Elcorock Containers**

The 0.75m³ Elcorock® Containers with a 'self-healing toe' were incorporated to reduce the risk of erosion undermining the foundations of the primary 2.5m³ Elcorock Container defence system. This system was considered to be a cost effective means of protecting the foundations of the main erosion defence system from the potential hazard of significant erosional events and to minimise the founding depth of the 2.5m³ Elcorock Containers.

Due to Contractor's resourcing constraints during construction, the 0.75m³ containers with a special 'self-healing toe' could not be used and the smaller bags wrapped in Elcomax® 600R high strength geotextile was used as per Figure 5 below.

Figure 5 – Alternative system to 0.75m³ self-healing toe

7.5 **Sloping Ground In front of the Gravel Raft**

The Elcorock® containers were only utilised along the seaward side of the gravel raft which was the area of greatest risk but with potential for extension and/or being raised in the future as required. To minimise damage from wave action to the main gravel raft and overlying slab along the other edges, engineered hard fill was placed around the edge of the gravel raft and gradually sloped away from the gravel raft to help gradually dissipate the energy from the storm waves as it approached the reinforced gravel raft. The hardfill was also required for landscape purposes.

7.6 **Underfloor Service Resilience**

In line with MBIE Guidelines, all underfloor services were structurally connected to underside of the reinforced concrete slab and flexible connections used at exit/entry to the slab and gravel raft.

7.7 **Construction Considerations**

The following observations during construction of the Elcorock® Containers warrant general comments for future schemes:

- The hopper attachment for 2.5m³ container was not sufficiently strong to hold a full sand container during placement and it was difficult to completely fill the container. The weight of 2.5m³ containers therefore varied and where used as 'sleeping defences' such as at Sumner, it could be advisable not to have structures/traffic on top of the container and area above is best left for general landscaping. It is noted that sand placement of the 0.75m³ containers was generally easier to control probably due to smaller size and different loading frame used.
- Site seams need to be orientated away from the seaside to prevent damage from the waves and this is particularly important for the 2.5m³ containers whose site seams/ends are hand stitched/woven. Seam orientation is also important during coordination phases of the project since the containers have different width and length hence 'erosion wall' footprint depending on container orientation.

8 CONCLUSIONS

While piled foundations have been historically used in coastal zones and generally recommended by some government agencies, there are circumstances where they do not provide the most cost effective and resilient solution particularly where anchoring into rock is required and/or where there are liquefaction induced hazards. Piled foundation are also generally not easily repairable following a damaging earthquake and shallow foundation options should therefore be considered provided all associated risks are understood and mitigated.

At Sumner Surf Life Saving Club, a composite shallow foundation system developed for replacement building has been discussed which is resilient and cost effectively addresses both the coastal and liquefaction induced hazards. The concept of incorporating a 'sleeping seawall defence' at design stage has been illustrated which can be incorporated in areas where future risk could occur but current circumstance do not warrant extensive mitigation measures.

It is noted that such systems, like any other foundation system, need to be considered on a case-by-case basis and may not be cost effective in some cases such as where extensive dewatering may be required during construction and/or active tidal zones. In such cases, piled structures are likely to be required but need to be appropriately designed to provide post-earthquake resilience in earthquake prone areas.

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